AGS VICTORIA 2016 SYMPOSIUM

Excavations and slope stability in Melbourne geology: experiences and recent developments

Wednesday, 16 November 2016, 12:00pm – 7:00pm
Engineers Australia, 600 Bourke Street, Melbourne

AUSTRALIAN GEOMECHANICS SOCIETY
VICTORIA CHAPTER

WELCOME

The Victorian chapter of the Australian Geomechanics Society (AGS) is pleased to welcome you to this half-day symposium titled “Excavations and slope stability in Melbourne geology: experiences and recent developments”.

Since the publication of the “Engineering Geology of Melbourne” in 1992, both the geotechnical profession and Melbourne has undergone significant change. Urban sprawl over the past few decades has seen increasing development in the hillside areas in the Dandenong and Mornington Peninsula regions. This coupled with changes to the regulatory environment and the introduction of the Landslide Risk Management Framework by the AGS in 2007 has changed the way in which local and state government as well as geotechnical practioners manage and assess slope stability.

In addition to development in hillside areas, significant development in the inner parts of Melbourne has posed many challenges for excavations not just in the soft soils of the Yarra Delta but also the weak rock of the Melbourne Formation.

This symposium seeks to bring together practioners from consulting, construction and academia to share and discuss their experiences on the separate, but related, topics of excavation and slope stability. Best practices, case histories and innovative solutions for dealing with these challenges will be presented and discussed, with a particular emphasis on local geotechnical issues.

ORGANISING COMMITTEE*

Daniel King (Chair)
Clare Bridgeman (Co-chair)
Mahdi Disfani
Jie Li
Jeremy Barber
Rob Day
Richard Kaser

*a sub-committee of the AGS Victoria committee

TECHNICAL REVIEWERS

Mahdi Disfani (Technical Chair)
Chris Boyd
Stuart Colls
Chris Coulson
David Glover
Richard Kaser
Steven Lankshear
Bing Lee
Ramtin Tajeddin

GROUNDWATER AND DEEP EXCAVATIONS

R. A. Day[1], AECOM Australia Pty Ltd

[1]AECOM Australia Pty Ltd, 727 Collins Street, Melbourne, VIC 3008 PH +61 3 9653 1234 email: rob.day@aecom.com

ABSTRACT

The presence of groundwater in deep excavations in soil and rock has the potential to profoundly impact both the stability of the excavation, and the impacts that the excavation can have on the areas surrounding the excavation. This paper is intended as a guidance based on the author's experience to some of the key considerations and pitfalls that designers of deep excavations should take into account when designing deep excavations where groundwater is present. The paper focuses on the influences of Melbourne geology and hydrogeology on the design of deep excavations and provides a brief summary of some of the important groundwater features of the central Melbourne area, but the principles discussed have wider application to projects in similar geology. A series of generalised examples are presented to demonstrate some of the important impacts that groundwater can have on the stability of excavations and structural loads on buried structures, as well as some of the impacts that dewatering activities can have on areas surrounding the excavation.

Keywords: groundwater, excavation, permeability, Melbourne, aquifer

1 INTRODUCTION

This paper commences with a brief recap of the geology of Melbourne's Yarra Delta as an understanding of its complexities which is key to understanding hydrogeological issues. The second part of the paper discusses the hydrogeological parameters and constraints that need to be taken into account when setting up models for assessing the stability and drawdown associated with deep excavations below the groundwater table. This part of the paper focuses on properties of some of the main aquifers and aquitards in the central Melbourne area and provides some general examples to illustrate the importance of these parameters. The third part of the paper works through some specific design examples to illustrate design considerations that should be taken into account in the design of deep excavations.

2 A BRIEF RECAP OF MELBOURNE GEOLOGY IN THE YARRA DELTA

An understanding of the evolution of the Yarra Delta is key to understanding groundwater issues associated with deep excavations in and around the central Melbourne area. An excellent starting point for understanding this evolution is contained in Peck et al (1992) which includes information from the Melbourne Underground Rail Loop and the Westgate Freeway amongst its sources. Since that time investigations and construction for major tunnelling projects has developed the understanding of this evolution. Some of the more important references documenting this development include: Hutchison & Lamb (1999); Ervin et al (2006); Leonard (2006); and Paul et al (2014).

It is not the author's intent to reproduce the information contained in these and other references, and the reader is urged to become familiar with these references before contemplating deep excavations below the water table in Melbourne. However, the following are some of the key points in the evolution of the Yarra Valley that have a significant impact on groundwater in deep excavations that are pertinent to the topics in this paper. For convenience a glossary of some of the hydrogeological terms used in this paper is included at the end of the paper. The following assumes the reader is familiar with Melbourne and its surrounds. The reader should refer to the Geological Survey of Victoria 1:63,360 Series Melbourne Map Sheet for the features, plan extents and typical section through the units described below:

- The basement rock beneath central Melbourne is the Silurian aged Melbourne Formation. This interbedded sandstone and siltstone has been folded into a series of synclines and anticlines trending NNE to SSW. The folding in some locations has resulted in bedding dips of steeper than 70^{o}. Superimposed on this folding is a system of faults and intrusion by a series of igneous dykes. The resulting fractured rock mass can be considered an aquifer of variable permeability. The Melbourne Formation has been uplifted and outcrops at the surface over most of Melbourne's CBD extending north towards Carlton, as well as in the hills south of the Yarra including the Domain, Botanical Gardens and South Yarra. South and west of this point, the Melbourne Warp results in the surface of the Melbourne Formation being less uplifted, dropping off towards the south west. Near the mouth of the Yarra River the rock surface is around 80m below sea level.

- There is a major unconformity between the weathered surface of the Melbourne Formation and the overlying Eocene and younger (formerly Tertiary) units. The older of these Tertiary units including the Werribee Formation and Newport Formation comprise interbedded sands silts and clays. These units do not typically outcrop in the central Melbourne area. They include both aquifer and aquitard layers depending on clay content.
- Interspersed with the deposition of the Werribee Formations and Newport Formation was a series of olivine basalt lava flows of the Older Volcanics. Although these basalts in some locations have weathered to a bouldery clay soil, immediately after deposition they were a hard rock resistant to erosion hence protecting the underlying silts and sands from being eroded by subsequent river activity. The result is that Older Volcanics outcrop at the surface at the western end of the CBD and parts of North Melbourne. These Older Volcanics also flowed down the paleo valley of the Yarra River splitting the valley into two forks through South Melbourne and Port Melbourne. (Note that some of the less weathered basalt formerly identified as part of this unit in the vicinity of the Swan and Swanston Street Bridges has recently been classified as the Swan Street Basalt of Quaternary age (about 1.6Mya).
- A disconformity after deposition of the Newport Formation occurred during which time much of the sediments deposited over the Older Volcanics were eroded away. Over the top of this eroded surface Brighton Group sand, gravels and clays were deposited in the Pliocene (late Tertiary). The Brighton Group forms a relatively thin cap over the Older Volcanics and Melbourne Formation in areas of Carlton, North Melbourne, the western end of the CBD and through to Toorak, but south of the Melbourne Warp, the Brighton Group becomes the dominant geology for many of Melbourne's south eastern suburbs.
- Between 1.1 and 0.9Mya significant erosion during several glacial minima occurred carving the paleo Yarra River into a wide fast flowing river valley. At the end of this period the Moray Street Gravels, a high energy river sand and gravel, were deposited in the base of the valley along with some earlier colluvial and alluvial sediments. A complicating part of this is that in at least some locations an early Quaternary basalt lava flow (Swan Street Basalt referred to above) pre-dates and underlies the Moray Street Gravels. This Basalt preserves earlier alluvial sands and gravels beneath it.
- Following deposition of the Moray Street Gravels sea levels rose rapidly and extensive depths of Fisherman's Bend Silt were deposited. The lower parts of this formation contain higher proportions of sand and may be relatively permeable while the upper parts deposited in a marine environment are predominantly a clayey aquitard confining the underlying Moray Street Gravel aquifer.
- After deposition of a significant depth of Fisherman's Bend Silt had occurred a period of multiple basalt lava flows of the Newer Volcanics occurred. Upstream of Melbourne CBD the Burnley Basalt Flow flowed down the Yarra River valley effectively creating a cap over the Fisherman's Bend Silt and forcing the Yarra River to form a new course further to the south and east on its current alignment. It is worth noting that at the time of European settlement a localised outcrop of basalt occurred across the Yarra River in the vicinity of Queens Bridge forming rapids with a fresh water river upstream of this point. This was removed using dynamite in 1883. Separate basalt flows of the Newer Volcanics occurred from the western suburbs forming the current western bank of the Maribyrnong River. There is evidence to suggest that Fisherman's Bend Silt continued to be deposited between and after these Newer Volcanics lava flows as Fisherman's Bend Silt overlies basalt in some parts of the Maribyrnong River. By the end of the deposition of the Newer Volcanics, basalt and the highly reactive clay soils resulting from weathering of basalt had become the dominant geology for many of Melbourne's western and inner north eastern suburbs. Jointing in the rock means that it is a relatively high permeability aquifer, although the weathered clay profile is of low permeability and forms a surface aquitard.
- Over the subsequent 0.6My prior to the Holocene there was relatively little major deposition of materials in the central

Melbourne area other than deposition of alluvium in creek and river valleys accompanied by erosion and weathering. Rapid sea level rise commenced at the end of the last glaciation 10,000 years ago. This sea level rise resulted in the deposition of Coode Island Silt in the valleys formed by the paleo Yarra and Maribyrnong Rivers and Moonee Ponds Creek. It extends under large parts of the Yarra Delta beneath areas of Kensington, North Melbourne, West Melbourne, Southbank, Docklands, South Wharf, South Melbourne and Port Melbourne as well as extending further upstream in the paleo valleys. Boreholes in the Port Melbourne area have identified Coode Island Silt over 20m thick with the deepest identified channel extending to 27m below sea level. Despite its name, Coode Island Silt is predominantly a high plasticity normally consolidated clay prone to consolidation settlement and an aquitard. However in the base of the channels into which the Coode Island Silt was deposited, Holocene aged alluvium had already deposited and forms a minor confined aquifer beneath the Coode Island Silt.

- In the latter part of the Holocene, Port Melbourne Sand was deposited as a dune system over the top of the Coode Island Silt over most of the Port Melbourne Area where it forms the surface soils. In parts of Port Melbourne and South Melbourne it extends to a depth of 10m to 15m below sea level and abuts open water along the Yarra River and Port Phillip. It is therefore an unconfined aquifer of high permeability.
- Prior to European settlement of Melbourne, the West Melbourne area was a tidal estuary with Coode Island Silt extending to the surface. Realignment of the Yarra River in the 1880s along with subsequent dredging and reclamation to form the docks has resulted in 2m to 8m thickness of fill placed over the Coode Island Silt to form the current surface. Fill materials are inherently variable and this introduces the potential for perched aquifers within the fill. Additionally the placement of fill surcharge over the Coode Island Silt often removed any slight over-consolidation in the Coode Island Silt and has resulted in long term consolidation settlement of the Coode Island Silt that has continued for decades. Any additional changes in surcharge pressure on the Coode Island Silt, whether due to filling or lowering of water pressures results in further consolidation.

3 HYDROGEOLOGICAL CONSIDERATIONS

When designing deep excavations, consideration needs to be given to the impact that groundwater pressures have on the stability of the excavation and the excavation support elements. Of equal importance is prediction of the volumes of groundwater inflow into the excavations that needs to be managed both in the temporary construction and long term final condition. This in turn causes changes to groundwater levels that impact on settlement and beneficial groundwater use in the areas surrounding the excavation. Some of the key hydrogeological considerations are discussed in the following sections:

3.1 Geomorphology

As discussed in Section 2, the evolution of the Yarra Delta is an excellent example of the importance of historical geomorphological processes to predictions of groundwater flow and drawdown. This evolution has resulted in a series of aquitards including the Coode Islands Silt, Fisherman's Bend Silt, higher clay content Tertiary sediments of the Werribee Formation, Newport Formation and Brighton Group and some weathered clays in the Older and Newer Volcanics. These aquitards create confined aquifers in the base of paleo channels as well as within layered sediments (e.g. Moray Street Gravels, Holocene Alluvium and other Quaternary channels, as well as older Tertiary aquifers including sandier layers of the Werribee Formation and Brighton Group, the Swan Street Basalt and less weathered rock of the Older Volcanics). Beneath all of this is the Melbourne Formation aquifer, and over the top of this, permeable unconfined aquifers are present in the Port Melbourne Sand and Newer Volcanics. Finally perched water is present at shallow depth in fill materials, in the weathered clay profiles near the surface of Older and Newer Volcanics and in the upper sandy layers of the Brighton Group.

The consequence of the above is that dewatering of one or more aquifers is likely to have an influence on water pressures in other aquifers and the extent of interaction between aquifers may be difficult to predict.

Although the above discussion is specific to Melbourne, elements of this type of geomorphology can be seen in coastal deltas around the world. Other geological environments will produce their own unique systems of aquifers and aquitards that need to be understood in order to design deep excavations.

3.2 Boundary Conditions

When modelling the impact of excavations on groundwater flow and drawdown, the initial tendency is to adopt simplistic boundary conditions that are relatively easy to model. The most commonly used boundary conditions are fixed head, no flow and fixed flow boundaries. However, in practice aquifer systems are rarely this simple. One method sometimes used to try and avoid making assumptions about boundary conditions is to model the boundaries a long way from the excavation at a distance believed to be beyond the zone of influence of the dewatered excavation. This logic can be acceptable for short term transient flow and radial flow towards a relatively small excavation, as the influence of boundary conditions drops off logarithmically at increasing distance from the excavation. However, for long narrow excavations such as for road cuttings or tunnels that are relatively large compared to the extent of the aquifer this can lead to misleading results particularly if steady state predictions are used. This is explained in more detail in Section 4.4, but as a simple illustration, consider the problem of a very long narrow trench excavation penetrating near full depth in an unconfined aquifer with a horizontal fixed head recharge at a distance D from the trench on either side as shown in Figure 1.

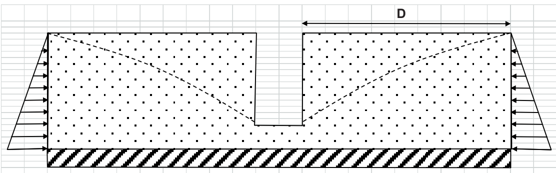


Figure 1: Fixed Head Boundary Example

Doubling D in the above example will approximately halve the rate of inflow to the trench and increase the drawdown extents in proportion to the distance to the boundary. With increasing distance D this becomes less and less representative of the real world as the further away boundaries are placed from the excavation, the more important is the consideration of recharge infiltration from surface water and outflow through extraction and deep infiltration to predictions.

Perhaps the closest approximation to fixed head boundary conditions in the Yarra Delta is the Port Melbourne Sand aquifer. It is directly connected to the Yarra River and Port Phillip which can provide an endless supply of sea water to horizontally recharge any dewatered excavation below the water table on the three sides of the unit abutting open water. Standpipe piezometers installed in this unit within a couple of hundred metres of the Yarra and Port Phillip have shown tidal variations confirming this connectivity. However, salinity in the Port Melbourne Sand, while high, is typically a fraction of that of seawater suggesting that infiltration from rainfall and runoff and leaky services is significant. Overall this results in slight mounding of groundwater levels in the middle of the Port Melbourne area with less saline water flowing from the middle of Port Melbourne towards the sea and river.

Much of the Port Melbourne Sand aquifer is underlain by Coode Island Silt which is of sufficiently low permeability that it can be considered a no flow bottom boundary with respect to groundwater flow rates, but not with respect to the impacts of drawdown on consolidation settlement as will be discussed later in this paper. A deep dewatered excavation in the Port Melbourne Sands will tend to reverse this direction of groundwater flow and suck large volumes of seawater back towards the excavation (i.e. saline intrusion). As well as introducing a groundwater disposal problem this may impact on beneficial groundwater uses such as the availability of suitable groundwater for vegetation.

By contrast to the Port Melbourne Sands where water levels are predominantly controlled by sea level supplemented by surface infiltration, groundwater levels in the Melbourne Formation and Newer and Older Volcanics where they outcrop in the hills of central Melbourne are predominantly as a result of surface infiltration, with groundwater flow occurring towards the lower lying river valleys. Assuming fixed head boundary conditions close to the excavation for this situation would tend to significantly over-estimate steady state groundwater inflow rates and underestimate the extent of the drawdown cone around excavations. Historically most deep basement excavations in the Melbourne CBD area have been in higher hill areas where the groundwater is deeper. These excavations have tended to extend only a few metres below the water table and have adopted permanently drained excavations. While these initially can encounter groundwater inflow, the tendency is for the flow to reduce or even dry up with time as the influence of the basement excavation permanently lowers the groundwater level in the surrounding area. In practice this has not caused major impacts as the Melbourne Formation is reasonably incompressible so settlement due to dewatering is negligible, and beneficial uses of the groundwater tend not to be impacted within the CBD. However with increasing depths of basements, particularly in lower lying areas below sea level, the potential for dewatered excavations to continuously generate groundwater and impact areas of Coode Island Silt prone to settlement along the Yarra River increases.

3.3 Permeability and Anisotropy

Permeability is a parameter with one of the highest natural variabilities we use in engineering. A high plasticity clay can commonly have a permeability of $1x10^{-8}$ m/s or lower, while a coarse clean gravel can have a permeability of 1 m/s or higher, a factor of one hundred million times higher than the clay. Under a constant head recharge and assuming that water can continue to recharge at this head, the amount of groundwater inflow to an excavation is directly proportional to the permeability of the ground. It is therefore very difficult to accurately predict inflow to excavations unless site-specific large-scale permeability testing is carried out in advance of dewatering activities. Soils and rock also typically have some level of anisotropy in permeability due to stratification and defects and this can also have a big impact, not just on the volume of inflow to excavations but in the lateral extent of drawdown caused by dewatering.

3.3.1 Soils

Some of the soils typically encountered in deep excavations in Melbourne and the implications of their permeability and anisotropy are discussed as follows. Not all units are discussed:

<u>Port Melbourne Sand</u> was deposited as coastal dunes and as a result it is typically fine to medium grained, clean sand of loose to medium density. Cross bedding with shells is observed deeper in the unit but at shallower depths shells and cross bedding are less apparent. Based on the limited cross-bedding, anisotropy would be expected to be relatively low. Leonard (2006) suggests permeability from pump tests in the range 1 to $5x10^{-4}$m/s, and salinity is generally in the range 2 to 10g/L (compared with 35g/L for seawater).

<u>Coode Island Silt</u> investigations for infrastructure developments have shown that although geologically classified as a single unit of Holocene age, it's engineering and hydrogeological properties vary considerably across the area of deposition. In deeper deposits particularly Port Melbourne, Coode Island Silt was deposited in an estuarine and shallow marine environment. As a result, in these areas it is a normally consolidated high plasticity clay with relatively little layering. Day and Woods (2007) back calculated permeability and anisotropy in a deep deposit of Coode Island Silt from an instrumented reclamation where settlement and excess pore pressure with and without wick drains was monitored. This indicated that vertical permeability averaged $2x10^{-10}$m/s (corresponds to coefficient of consolidation c_v of $0.4m^2$/year) with horizontal permeability between 3 and 5 times higher than vertical permeability without wick drains or approximately equal to vertical permeability with wick drains. The author is aware of two other reclamation projects in deep Coode Island Silt that gave similar results.

By contrast around the fringes of its deposition area, particularly Kensington, North Melbourne, parts of Southbank and further up the paleo valleys, Coode Island Silt less than 5m thick is common. In these areas, some investigations have reported Coode Island Silt generally of a lower plasticity and higher sand content containing numerous sand and shell beds consistent with a shallow estuarine and coastal deposition. Rates of consolidation in these parts of the Coode Island Silt have anecdotally been much more rapid. While more rapid consolidation would be expected anyway due to the thinner total thickness, rates of consolidation suggest permeability may be one to two orders of magnitude higher than in the Port Melbourne area. Additionally Srithar (2010) and Ervin et al (2006) suggest that the horizontal permeability may be 10 to 100 times the vertical permeability due to sand layers.

Where these sand layers can freely drain they have the potential to further reduce drainage path lengths and increase rates of consolidation. Deep excavation below sea level in the Coode Island Silts around Southbank for Hamer Hall and the State Theatre in the 1970s resulted in dewatering of the Coode Island Silt causing consolidation settlement damage to some surrounding buildings. Perhaps as a result of this, subsequent building developments in the Southbank area tended to avoid basement excavations below the water table.

During construction of CityLink's Southbank Interchange circa 1999, excavation below the water table in Coode Island Silt was required. As reported in Ervin et al (2006), the presence of sand layers increasing the horizontal permeability of the Coode Island Silt introduced the potential for groundwater drawdown to affect relatively large areas of Southbank around the excavation. To prevent this occurring a bentonite cut-off wall and shallow recharge wells were installed around the excavation. In more recent years a small number of buildings in the Southbank area have started to adopt basement excavation below the water table again. At least one of these has used the bentonite cut-off method to control the extent of drawdown induced consolidation in surrounding areas.

<u>Fisherman's Bend Silt</u> has been dredged and excavated extensively to form the various channels and port facilities on the Yarra, but dewatered excavations into this unit to date are rare. Information on the permeability largely comes from reclamation settlement monitoring. Because the Fisherman's Bend Silt is more overconsolidated than the Coode Island Silt the magnitude of settlement is usually smaller and as a result there is less monitoring carried out and permeability is harder to accurately determine from settlement monitoring. Laboratory testing and limited settlement monitoring suggests that permeability of the upper Fisherman's Bend Silt is of a similar order of magnitude to the Coode Island Silt. It is likely that higher permeability layers occur particularly in the lower Fisherman's Bend Silt and this is likely to result in high anisotropy in the unit as a whole.

<u>Holocene Alluvium</u> is believed to be a relatively narrow and thin layer of sands and gravels deposited in the base of the paleochannels that existed prior to the deposition of Coode Island Silt. It is of significantly higher permeability than the overlying Coode Island Silt or the underlying Melbourne Formation and Fisherman's Bend Silt (depending on location). The effect of this is to produce a narrow confined aquifer that does not have a ready source of recharge other than seepage from the surrounding lower permeability soils and rock. Any dewatering of this aquifer can therefore quickly drain the aquifer for significant distances along the length of the aquifer. An example of this effect is discussed in relation to the Moray Street Gravel below.

<u>Moray Street Gravels</u> were deposited in a narrow valley upstream of the CBD but the valley widened downstream of this point resulting in more extensive deposits beneath much of the Port Melbourne Area. They are a confined aquifer overlain by Fisherman's Bend Silt and underlain by either the Melbourne Formation or the Werribee Formation depending on location. Like the Holocene Alluvium, dewatering of this aquifer can potentially drain the aquifer for significant distances. An important distinction however is that the volume of the Moray Street Gravel is much larger than the Holocene Aquifer hence a larger volume of water needs to be extracted to lower water levels, and its larger plan extents means that although recharge rates from infiltration through the Fisherman's Bend Silt are slow they can still provide some recharge to the top of the Moray Street Gravel making it behave as a leaky confined aquifer.

Hutchison and Lamb (1999) have documented that during construction of Melbourne's CityLink tunnels in the Melbourne Formation, dewatering of the Siltstone in the tunnel excavations to up to 60m below sea level resulted in depressurisation of extensive areas of the Holocene Alluvium and Moray Street Gravel aquifers where they were in contact with the Melbourne Formation with a drawdown of to 10m measured at distances of up to 1.5km from the area of maximum drawdown in the tunnel. This in turn led to consolidation settlement of the Coode Island Silt overlying these aquifers. To compensate for this, temporary recharge was required in the aquifers during construction.

<u>Brighton Group</u>

Considerable care needs to be given to assessing permeability properties of the Brighton Group. It is made up of interbedded clayey sands, sandy clays, sands and gravels. As a result a very high level of anisotropy within the unit as a whole can be expected. The horizontal permeability relevant to an individual excavation will also depend on whether or not a significant sand or gravel layer is encountered, and whether if encountered it is reasonably continuous and connected to other high permeability layers.

Leonard (2006) reports transmissivity of some of the higher yielding bores in aquifers of the Brighton Group which suggests a horizontal permeability of around 1×10^{-5}m/s. However, it also reports transmissivities which suggest permeability at least an order of magnitude lower than this. Anecdotally, excavations several metres below the water table in some locations have produced very little groundwater suggesting permeability of possibly 100th of this permeability, while in other locations flow from sandy layers within the Brighton Group has caused bored pile and localised excavation collapse suggesting a high permeability.

An added complication when comparing results from pumping of the Brighton Group is the variability with location. In the western suburbs it is a confined aquifer beneath the Newer Volcanics and may not be continuous. In the south eastern bayside suburbs it is underlain by the Fyansford Formation which is considered a significant aquifer and the two units are often considered together as one aquifer. In these locations Brighton group is both an unconfined and semi-confined aquifer. In the inner northern suburbs and higher hills of the inner south east, Brighton Group outcrops at the surface significantly above the regional groundwater table. Groundwater in these locations tends to be perched and therefore also tends to dry up when dewatered.

Leonard (2006) also reports a wide range of groundwater salinity in the Brighton Group ranging from <0.3 to 8g/L (ignoring an area around Point Cook beneath salt crystallisers). This also points to the variability in confinement and sources of recharge with location.

Historically there have been relatively few deep excavations below the water table in the Brighton Group on which to make these assessments, but in recent years deep excavations have started to occur and can be expected to increase into the future, particularly for high rise developments and rail projects along St Kilda Road and inner south eastern bayside suburbs.

3.3.2 Rocks

In most rocks, defects, particularly bedding, joints, faults and karst are the most important factor affecting permeability and anisotropy. Figure 2 shows that a clean sand with 10% fines passing the 75micron sieve might typically have a permeability of $1x10^{-4}$m/s. By comparison an otherwise low permeability rock has the same equivalent permeability in the direction of defects if there is a 0.2mm wide open continuous joint or a single 4mm diameter "pipe" defect per square metre of area. This demonstrates not only that the permeability of a rock mass is very sensitive to the thickness, persistence and interconnectivity of rock defects, but also that the orientation of defects can result in high levels in anisotropy in rock masses.

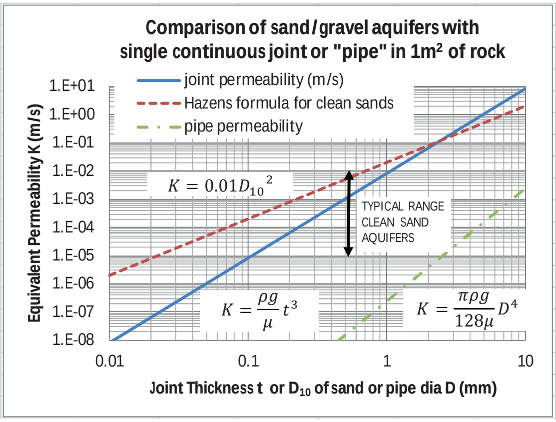


Figure 2: Effect of defects on permeability

In addition to consideration of the persistence and interconnection of defects in rock masses, is consideration of the level of infilling or opening of defects due to the effects of weathering, alteration and erosion. For example rocks such as limestones which may be tight jointed in a fresh unweathered state can be full of solution cavities when subjected chemical alteration, dissolution or weathering. Volume changes from these processes can also lead to opening of fractures. By contrast volcanic rocks that can be open jointed in a fresh state can have the joints infilled with clay in a highly weathered state.

In the Melbourne Formation there are typically three major defect sets: bedding and two major joint sets roughly orthogonal to bedding. Superimposed on this the rock has been heavily folded and faulted and intruded by igneous dykes. The resulting rock mass can have a high variability in permeability with nearly all of the flow occurring along defects. Bedding has a high level of persistence and can be expected to produce a high level of anisotropy in the permeability of the rock mass, but faulting, especially where there has been significant movement and fracturing on the fault can be even more persistent and can be a dominant flow path for groundwater. By contrast igneous dykes weathered to clay can provide a low permeability cut-off to flow.

Hutchison and Lamb (1999) identified from back-analysis of CityLink inflows that the Melbourne Formation had an "average permeability of $2x10^{-6}$m/s, although much higher in certain geologic structures". The author has observed that groundwater inflows from the rock are almost entirely from discrete defects in the rock. It is possible to drill one hole several metres into rock that produces virtually no inflow while another borehole as little as 1m away can produce a significant inflow if it encounters permeable defects.

In basalt of the Newer Volcanics, joints formed as the basalt cools can be very wide, persistent and interconnected resulting in very high permeability. However, as described in Leonard (2006), the Newer Volcanics are a multi-layered fractured rock aquifer system with sheet-like basalt aquifers separated by thin clay layers of low hydraulic conductivity. The uppermost aquifer is largely unconfined; but lower aquifers are semi-confined to confined. Additionally weathering of the rock produces high plasticity clay and in highly to extremely weathered basalt the clay can infill the defects producing a much lower permeability rock mass. As a result the permeability and anisotropy of the Newer Volcanics is highly variable.

3.4 Storativity

Storativity in an aquifer identifies the volume of water yield due to a lowering of water pressure in the aquifer. In dense or tight fractured confined aquifers the yield is typically a very small percentage of the soil volume, but in an unconfined aquifer with a high degree of connected voids such as the Port Melbourne Sands the yield from lowering the water table can be 30% or more of the soil volume dewatered. When predicting steady state inflow to an excavation storativity does not come into the calculations, but for transient groundwater flow, storativity can have a big effect on predictions of inflow rates and drawdown. For example the initial rates of inflow into a drained excavation in Port Melbourne Sand can be one to two orders of magnitude higher than the long term steady state inflow.

On the other hand, in overconsolidated clay or silt soils (for example clayier units of the Brighton Group), the yield from lowering the water table may be less than a couple of percent of the soil volume dewatered, but where these soils are also of low permeability the time taken for this water to drain from the soil is much slower. The implication of this is that while steady state drawdown predictions may extend for large distances around a deep excavation, a temporary excavation of a few weeks or even a few months duration may be controlled by transient conditions and the drawdown may localised to a relatively small area around the excavation. The time taken for this to occur becomes very sensitive to the relative permeability and storativity.

Normally consolidated clays (e.g. Coode Island Silt) have a high void ratio which can result in a high water yield per cubic metre of soil if lowering the water pressures results in consolidation. Because Coode Island Silt is near-normally consolidated, lowering of water pressures by more than 1 to 2m is typically sufficient to cause consolidation of the Coode Island Silt. However because of the low permeability of these soils the consolidation can take years or decades to reach steady state groundwater conditions.

3.5 Design Groundwater Levels/ Pressures and Changes with Time

The selection of design groundwater levels or piezometric pressures can have a dramatic effect on the design of an excavation retention system. Where the retention system is designed to structurally resist water pressures, a higher water pressure will often be the critical condition but this is not always the case as will be discussed in relation to tunnel linings later in this paper. Where the structure is drained, changes in water levels can impact prediction of inflow rates but can also cause other impacts including consolidation (as discussed above in relation to the Coode Island Silt) and more widespread effects on other structures and infrastructure and natural environment including the availability of groundwater for other beneficial uses.

Groundwater levels can change significantly with time in response to a number of factors including the effects of the excavation itself, but also due to seasonal effects, rising sea levels or rainfall due to climate change, storm and tide events, and the impacts of other infrastructure and development (e.g. construction of a dam or another deep excavation).

An interesting example of man-made increases in water level with time is the city of Riyadh in Saudi Arabia where an underground Metro rail project is currently under construction. Although a desert environment, the city is underlain by highly permeable limestone breccia and hence what water does fall on the catchment rapidly infiltrates the surface to recharge a regional water table beneath the city. The inland city gets the majority of its water supply from desalination plants on the coast pumped inland. The rapid growth of the city with resulting irrigation and leaky stormwater and sewer pipes has gradually raised the water table over a period of decades to the extent where groundwater dewatering bores have to be employed to prevent older basements that were dry at the time of construction in some parts of the city from being flooded with groundwater.

By contrast, investigations for both the Bangkok Metro and Perth Metro underground stations identified that deep extraction groundwater bores had lowered groundwater pressures to significantly below sea level in some parts of the subsurface profile.

In all three cases the design of the underground structures had to consider the potential for changes in extraction with time leading to increased or decreased water pressures.

In addition to long term changes in groundwater pressure, consideration also needs to be given to the impact of short term flooding events on excavations, particularly with respect to the possibility of perched water tables developing in permeable backfill materials against retaining walls.

3.6 Permeability changes with time

In addition to changes in groundwater pressures with time it is also important to consider permeability changes that can occur with time. In normally consolidated and lightly overconsolidated soils such as the Coode Island Silt and Fisherman's Bend Silt, consolidation due to surcharging as well as causing settlement will also result in a reduction in permeability of these soils.

The groundwater chemistry of many of Melbourne's aquifers including the Melbourne Formation and the Quaternary sediments can be quite aggressive. Exposure of groundwater to oxygen and cement products as well as changes in pore pressure can lead to precipitation of ions or chemical reactions which result in deposition of materials which can clog flow in both the soils and rocks and in the drainage systems installed to collect groundwater.

On the other hand, in infilled rock defects and silty sand and gravel aquifers, continued groundwater flow can erode or flush the fines leading to increases in permeability with time. This can be of particular concern in dispersive soils where groundwater flow can lead to progressive piping with rapid increases in permeability.

4 DESIGN EXAMPLES

4.1 Flexible Retaining Walls

Flexible retaining walls from a groundwater perspective can be divided into water-tight and permeable walls. Soldier piles with a drainage medium between piles often used in better rock and clay conditions in Melbourne are permeable. The following discussion relates to water tight secant pile, diaphragm and sheet pile walls used for permanent and temporary support of excavations in saturated permeable soil profiles. Some of the important geological profiles to consider for existing and future development in Melbourne are the Brighton Group sediments and the sediments of the Yarra Delta (particularly the Port Melbourne Sand, Coode Island Silt and Fisherman's Bend Silt and associated fill).

Water pressures on flexible retaining walls significantly increase the bending moments in the wall and significantly reduce the toe stability compared with a retaining wall in the same materials where the water table is below the toe of the wall. The means by which the excavation is dewatered during construction also has a significant impact on both the volumes of groundwater to be dealt with and loads imposed on the wall.

To illustrate this consider a hypothetical case of a two storey basement excavation 5m deep in Port Melbourne Sand (PMS) with the geometry shown on Figure 3. The walls are considered impermeable and are internally propped at one level such that kick-out of the wall toe is the geotechnical failure mode.

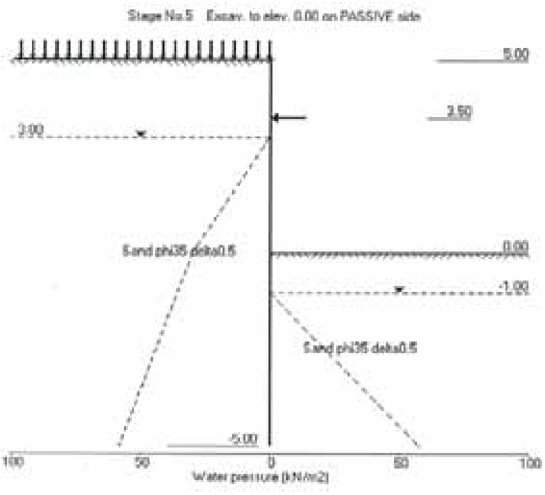


Figure 3: Example flexible wall in PMS

For the purpose of this example the initial water table is assumed to be at the surface and a single layer of uniform sand is assumed with no layering or anisotropy in the soil. A no flow boundary has been adopted 5m below the wall toe and a constant head recharge boundary fairly close to the excavation has been assumed. As will be discussed later, layering, anisotropy and boundary conditions can also significantly affect the result.

If the sand was dewatered to below the wall toe using dewatering spears on the outside of the wall, then the wall behaves essentially as a dry retaining wall. Under these conditions active pressure on the outside of the wall is low, passive pressure on the embedded wall toe is high and there is a high factor of safety on pile toe kick-out, hence toe penetration could theoretically be reduced to 2m. However this would require lowering of the water table by 8m resulting in large volumes of groundwater disposal and drawdown of the water table for a large distance around the excavation with potential settlement impacts.

It is typically more desirable to dewater from sumps in the base of the excavation allowing water to flow up through the base. The resulting pore pressure gradient shown in Figure 4 reduces the amount of inflow to the excavation and drawdown outside the wall is much reduced. However the impact of this is to increase water pressures on the outside of the wall. Additionally the upward pressure gradient dramatically reduces the effective weight of the soil in front of the toe and hence the passive resistance. As shown on Figure 5, the active pressure on the back of the wall is increased by 300% and the passive pressure in front of the wall reduced to 30% compared with a dry excavation. This significantly increases the moments in the wall, propping forces and toe penetration required to maintain stability.

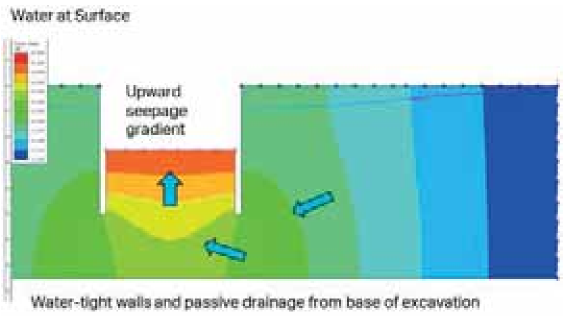


Figure 4: Passive dewatering of excavation

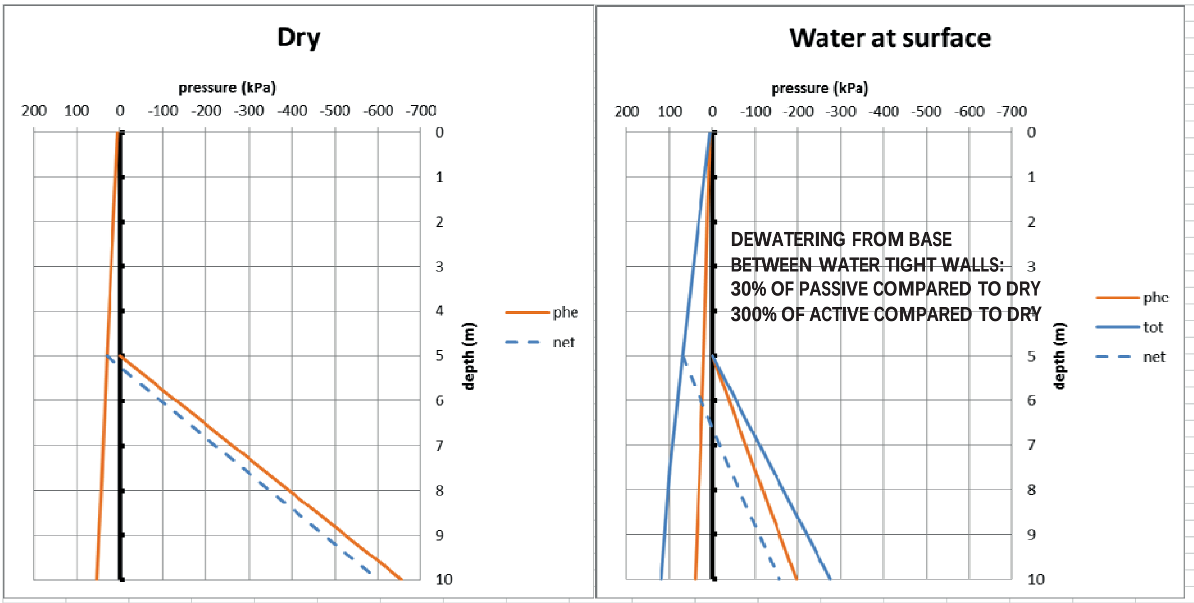


Figure 5: Comparison of earth pressures

4.2 Impacts of Anisotropy

The above example assumed isotropic conditions. As has been discussed in Section 3.3, interbedded sand and clay layers can result in high levels of anisotropy. To demonstrate the potential impacts, Figure 6 shows the changes in head and flow rates for the flexible retaining wall example from Figure 3 if an isotropic permeability of 1×10^{-6}m/s is increased by a factor of 10 in the horizontal direction. In this example the theoretical flow rate into the excavation increases by a factor of 2.5, the drawdown on the outside of the wall is reduced from 2m to zero, and the upward hydraulic gradient in the base of the excavation increases from around 1.5 times hydrostatic to around 2 times hydrostatic. The significance of this is that upward porewater pressure exceeds the self-weight of the soil in the base of the excavation. Under these conditions the soil in the base of the excavation undergoes uncontrolled heave and there is no passive resistance on the wall leading to collapse. Issues of base stability and buoyancy are discussed further below.

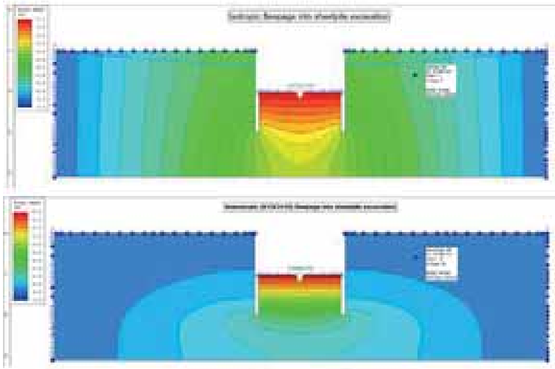


Figure 6: Effect of Kh = 10Kv

4.3 Base Heave and Buoyancy

The example shown in Figure 6 in the previous section showed how flow towards the base of an excavation produces an upwards hydraulic gradient in the floor of the excavation leading to base heave and instability. Depending on the type of ground conditions this can manifest in different ways. In sands, it is possible for “sand boils” to occur where water flow initially causes sand and water to bubble up at the surface. As this continues the water causes “piping” where sand is washed away locally to create higher permeability zones where water can flow relatively freely. This further increases the rate of flow and sand wash-out into the excavation and the problem gets progressively worse until either water levels are lowered outside the excavation sufficiently to cause equilibrium, or a collapse of the walls occurs. Where interbedded clay and sand layers are present causing anisotropy, it is possible for water pressures to build up in the sand layers and cause a mass heaving of the overlying clay.

To prevent these types of failures occurring pressure relief wells are one method to reduce the upwards hydraulic gradient in the base of the excavation. These are installed to sufficient depth and spacing that a near hydrostatic water pressure is produced below the base of the excavation. Unfortunately a side effect of this is to significantly increase the inflow to the excavation and increase the drawdown outside the excavation. Where large drawdown outside the excavation is not permissible due to environmental or consolidation settlement considerations, recharge wells outside the excavation may also be employed. However, this further increases the amount of inflow to the excavation. Additionally, if the recharge wells are placed too close to the dewatering wells, short circuiting of groundwater flow between the recharge wells and pressure relief wells can occur which re-introduces a piping problem.

As discussed in Sections 3.1 and 3.2, the geomorphology and boundary conditions at the site can also have a significant effect on base heave. Figure 7 shows an idealised example of an excavation at the base of a large hill or mountain to demonstrate this effect. The example shows a basement rock comprising Melbourne Formation with an upper weathered profile including residual clay soils. At the base of the hill is a Quaternary clay rich alluvium. This type of condition is quite typical of conditions in the north east of Melbourne. Rainfall on the hill slope raises the groundwater levels in the hill to well above the water level in the floor of the valley. The less weathered rock at depth is of relatively high permeability compared with the residual clays and as a result an upward pore pressure gradient is created in the valley floor. Near the base of the hill it is possible for artesian groundwater pressures to be developed in the fractured rock beneath the clay soils. Under these conditions a basal heave problem can be created even for a fairly shallow excavation.

Excavations that require recharge systems to control drawdown are typically not an acceptable long term solution. In these instances the long-term solution will normally be to adopt a water-tight or “tanked” permanent structure. This tanked structure will be subjected to significant buoyancy forces on the base of the structure. The design of the tanked structure for buoyancy needs to consider a range of possible groundwater pressures including allowance for future events.

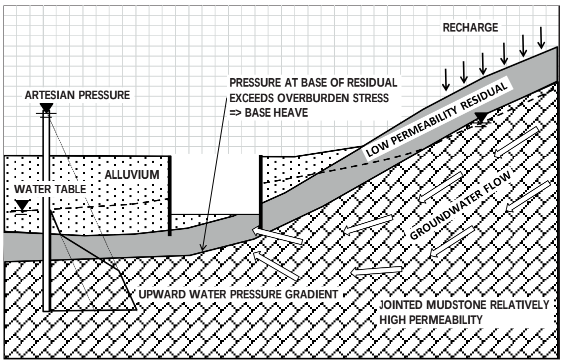


Figure 7: Basal heave at the base of a hill

Figure 8 shows an example typical of the conditions encountered for the Bangkok Metro project. Bangkok is located in a low-lying coastal river delta underlain by soft estuarine clays (similar to Coode Island Silt), underlain in turn by interbedded sands and clays (similar to

Melbourne's Tertiary sediments). To provide water for a growing population numerous groundwater bores have been installed into both the shallow and very deep sand aquifers and this has led to dewatering of the upper sand aquifers to depths of 20m to 30m below sea level. This in turn has caused consolidation of the upper soft clays to the extent that parts of Bangkok are below sea level and are flooded in the wet season.

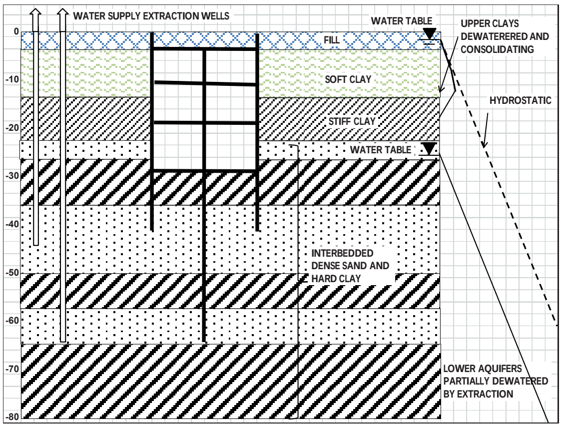


Figure 8: Effect of deep extraction wells

The long term groundwater management plan is to provide water from sources outside of Bangkok and restrict dewatering of the aquifers to prevent the flooding from getting worse. An advantage of the dewatered groundwater profile during excavation for the underground rail stations was that basal heave was not a significant issue during construction. However, the design had to consider the eventual reinstatement of a hydrostatic groundwater pressure together with a design flood event. The resulting buoyancy loads were much higher than the self-weight of the station and deep piles and diaphragm walls were required to help resist flotation of the stations.

Where short term flooding or other rare events results in buoyancy forces that are much higher than the structure would experience under normal conditions, another solution to prevent a catastrophic collapse under the much higher pressures that may occur under these low probability events may be to provide some form of positive pressure relief for pressures above a design pressure.

A common example of this is the design of underground water storage tanks, sewage treatment tanks and swimming pools which under normal operating conditions have enough weight in the structure to resist buoyancy, but when empty can float. In these instances pressure relief valves are sometimes fitted to the base of the tank to allow groundwater to fill the tank if it starts to uplift.

4.4 Impacts of Boundary Conditions

The examples to date have all considered a fixed head boundary at relatively close proximity to the excavation. As discussed in Section 3.2, this type of boundary condition is often not a realistic assumption and depending on how it is applied, can lead to unconservative results.

Consider the isotropic and anisotropic examples in Figure 6. With a fixed head boundary and isotropic soils, doubling the distance to the boundary approximately doubles the groundwater flow path and hence the flow rate approximately halves and the amount of drawdown outside the wall approximately doubles. However for the anisotropic condition nearly all of the head loss occurs due to the upward flow through the base of the excavation and doubling the distance to the fixed head boundary has almost no effect on the rate of flow or the drawdown outside of the excavation wall.

By contrast, if the fixed head boundary is replaced by a boundary condition which limits the flow rate to the water available in the catchment, the results can be much more sensitive to assumptions. As a simple example consider the isotropic soil example in Figure 6. In order for the fixed head steady state assumptions in this example to be valid there has to be a sufficient supply of recharge to the boundary to match the flow rate into the excavation. If the flow rate that could be supplied to the edge boundaries was say half of the flow required to maintain a fixed head condition, then the water table at the boundaries would lower by about half the total drawdown (about 2.5m) causing increased drawdown at all distances from the excavation and the flow rate into the excavation would reduce to match the available recharge (see Figure 9).

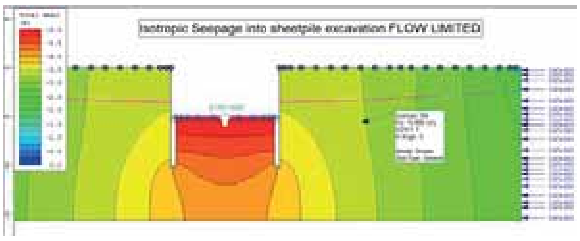


Figure 9: Effect of limiting boundary flow rate

Consider the example of Riyadh discussed in Section 3.5. The city is bounded by river valleys to the east and west of the city which effectively define the extent of the catchment. However most of the time these rivers are dry and as discussed, the principal recharge to the system is surface infiltration including infiltration from irrigation and leaky services. As part of the Riyadh Metro project, Lines 1 and 2 require excavation of 14 stations to depths of 5 to 20m below the water table. Estimates of initial inflow rates to these excavations if all of them were to be excavated simultaneously with no form of groundwater control were approximately 2 to 3 times the total estimated increase in storage to the aquifers causing regional rise in groundwater levels. The implication of this is that excavation of this type could potentially reverse the groundwater rise and cause large groundwater drawdown to large parts of Riyadh. This in turn would significantly reduce inflow rates to the excavations with time.

The example of drained basements causing permanent lowering of groundwater levels in Melbourne's CBD discussed in Section 3.2 is another case of inflows to excavations limited by the rate of infiltration into the catchment.

The example from Hutchison and Lamb (1999) discussed in Section 3.3.1 of depressurisation of the Holocene Alluvium and Moray Street Gravel aquifers by the CityLink tunnels demonstrates that for a large tunnel excavation there was insufficient recharge into these confined aquifers to keep them pressurised. However, for a small, localised excavation, for example excavations for repairs to a deep sewer, it is possible that these aquifers could have sufficient seepage through the overlying aquitards that they could behave similarly to a fixed head boundary condition.

The point to be made from the above examples is that when dealing with deep, large scale excavations, consideration needs to be given to the sensitivity of results to assumptions about boundary conditions. Depending on the results of sensitivity analysis it may be appropriate to analyse the impact of excavations using catchment level groundwater models that take into account transient conditions, regional infiltration and recharge sources, and the relationship between different aquifer and aquitards.

4.5 Rock Cut Stability on Defects

Deep excavations in Melbourne Formation below the water table for basements and road/rail cuttings and tunnelling are widespread in Melbourne and will continue to be a feature of future development. Overall the depth of these excavations can be expected to increase with time extending deeper below the water table. A key feature of the Melbourne Formation is that folding of the bedding can result in bedding dipping steeply out of one or more faces of excavations. Coupled with the two major joint sets roughly orthogonal to bedding this provide ample opportunity for release planes that can allow wedge or planar sliding on defects. Water pressure on these sliding planes can substantially impact stability. The difficulty in assessing the impact is deciding what water pressures can develop on the sliding plane compared with the water pressures applying to the rest of the rock block and the retaining structures. Transient water level rise can be a significant problem in rock slopes as fractures fill rapidly after rainfall with little recharge volume required. Water pressure can increase by many meters for very short periods of time and may be missed by conventional monitoring. Data loggers are needed to identify these short duration events.

For a rock excavation retained by a water-tight retaining wall, the loads on the retaining wall can be considered in the same way as discussed for flexible retaining walls. A groundwater table reduces the buoyant weight of the rock block reducing the force required to retain the rock block, but there is a significant increase in water pressure on the retaining wall that overall increases the retaining wall forces.

For an exposed rock face there is no water pressure at the excavation face, but the amount of water pressure on the sliding plane can vary considerably depending on the variability of permeability along the sliding plane and the potential sources of groundwater able to recharge the water in the joint. As shown on Figure 10, where joints in the rock are reasonably permeable and connected, it is likely that excavation will result in a lowered water table for a considerable distance around the excavation and water pressure on the sliding plane is likely to be low. For joints that connect into the sliding plane but do not necessarily daylight into the face except for the sliding plane itself, groundwater flow along the sliding plane can lead to higher pore pressures on the sliding plane particularly at its mid-length. At the extreme, where the sliding plane is more permeable at shallow depth with a source or recharge (e.g. ponded water or leaky service) and the sliding plane tightens towards the base of the excavation, near-hydrostatic water pressures can potentially develop.

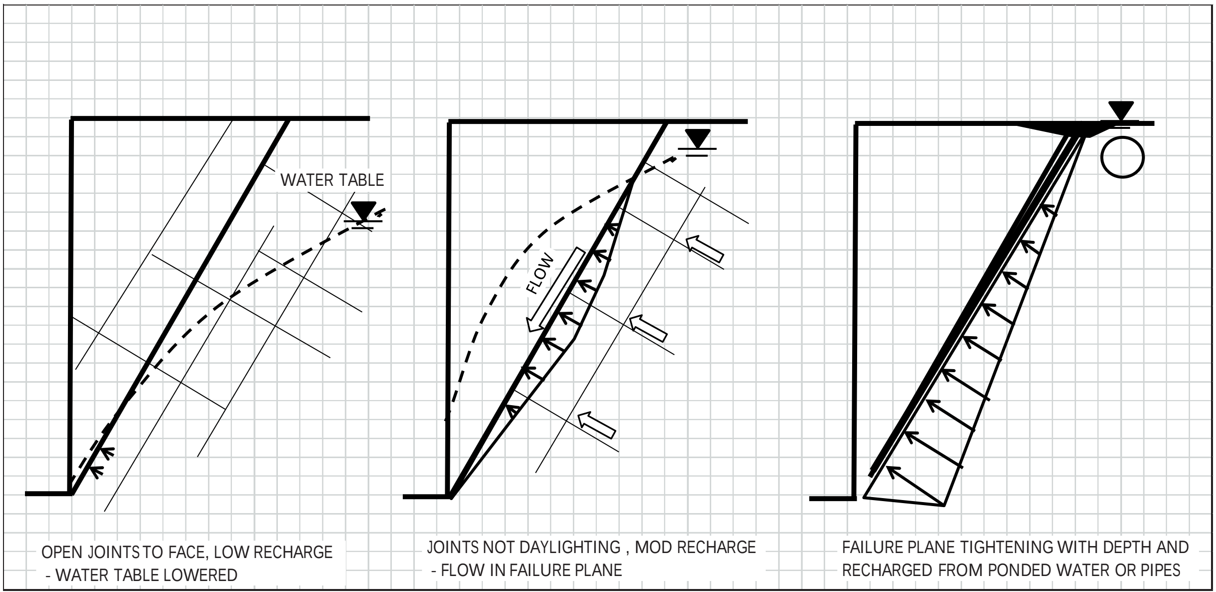


Figure 10: Comparison of earth pressures

To put this into perspective, using typical properties of Melbourne Formation, bedding dipping out of an excavation face at 60 degrees would require rock bolting even with no water table present, but if full height hydrostatic pressures developed on the sliding plane, the loads on the rock bolting would increase by around 60%. Alternatively bedding dipping out of the face at 27degrees with a factor of safety of 1.5 in a dry state could reduce to the point of instability with full hydrostatic pressure on the failure plane.

4.6 Tunnel Linings

Some important considerations when designing tunnel linings to resist water pressures that are sometimes overlooked include:

- The effects of buoyancy (difference in water pressure between the invert and obvert of the tunnel) can be significant for large tunnels;
- The worst stresses on the tunnel lining may occur at lower water pressures than the design maximum water pressure.
- Significant gaps between the tunnel lining and the rock can develop which can provide a path for groundwater flow along the length of the tunnel, particularly with drained, mined construction methods.
- Non circular tunnel linings can be subjected to significantly higher lining stresses than circular tunnels under high groundwater pressures.

These effects can be illustrated with an idealised example. For the purposes of the example a 15m diameter circular tunnel is excavated by road header into medium strength rock with the invert 60m below a water table as shown in Figure 11. In theory the rock has sufficient strength that a drained excavation into the rock can be carried out with minimal rock support. In practice the tunnel would likely be excavated in multiple headings with primary support comprising pressure relief drainage in areas of higher inflow, and rock bolting and shotcrete to stabilise wedge failures, but this primary support is ignored in the example.

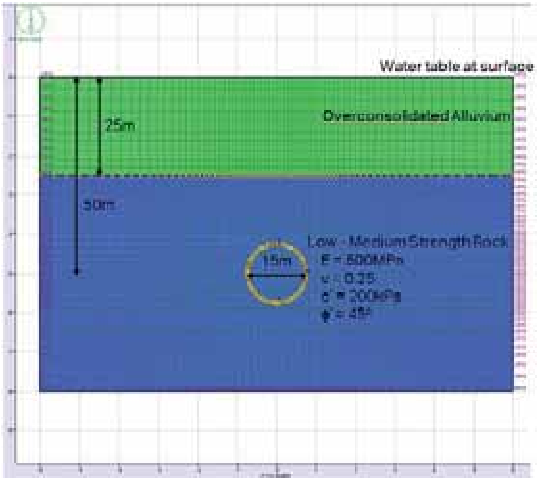


Figure 11: Idealised tunnel model

For a rock mass modulus of 500MPa there is an average of 20mm of radial rock movement at the tunnel lining due to excavation. Most of this movement is due to increase in circumferential rock stress caused by the excavation, but about 5mm of it is caused by the radial groundwater pressure gradient towards the excavation as shown in Figure 12.

A 0.5m thick secondary liner is then cast against the rock. In practice a waterproofing membrane would be placed against the secondary liner first and this is typically backed with a geotextile fleece to prevent damage to the liner. The liner is then typically cast using a slip form which is removed once the concrete has gained sufficient strength. At this stage the excavation is still drained. The fleece is compressible and additionally the concrete shrinks during curing. As a result there is a small gap formed between the rock and the liner. For the purposes of this example an initial gap of 5mm is assumed, although as will be shown below, a much bigger gap develops as the liner is subjected to water pressures, so the results are not particularly sensitive to the initial gap.

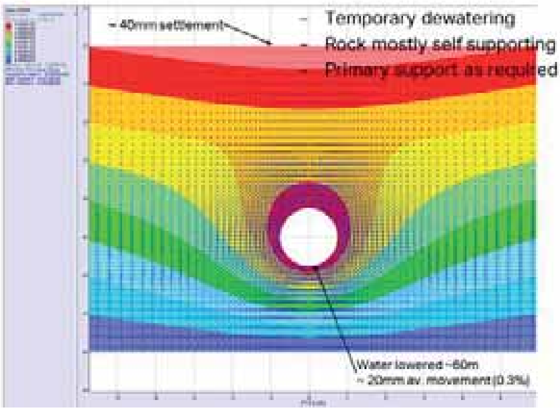


Figure 12: Drained excavation pore pressures

When the slip form is removed the lining needs to deform to support its own weight on the invert of the tunnel excavation. The result is that an additional small gap forms at the crown as shown in Figure 13.

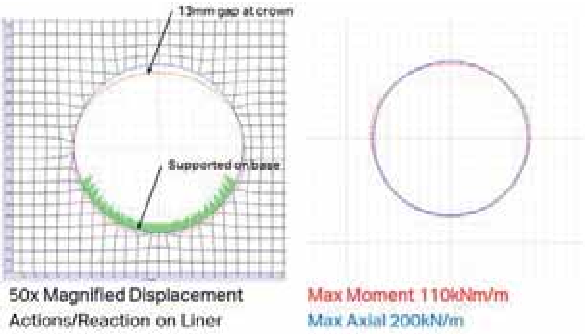


Figure 13: Self weight of lining

The tunnel lining may stay in this state for several months while the rest of the tunnel is completed before drainage of the excavation can be turned off and water pressure is allowed to recover.

As water pressures start to recover, one critical stage is as the water pressures rise to around the crown of the tunnel. At this point there is 150kPa of pressure at the invert of the tunnel and zero pressure at the crown and the full buoyancy force is applied to the tunnel lining. The tunnel lining "floats" to the top of the tunnel excavation and is held down by a reaction force from the rock above the crown. At this stage the compressive force in the lining together with the ovaling effect of the buoyancy creates a 25mm gap below the invert of the lining as shown in Figure 14. Because the bending moments from buoyancy forces are reasonably high but axial forces are still low, this is the critical stage for an unreinforced liner and dictates the need for a 0.5m thick lining.

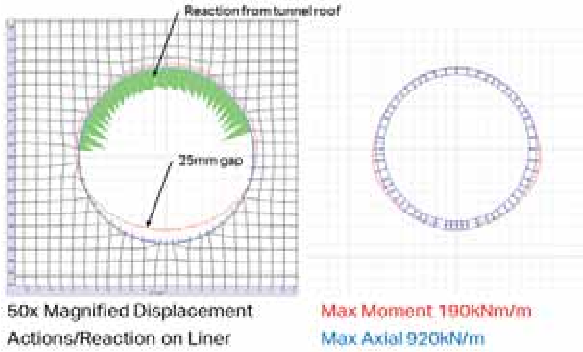


Figure 14: Water to crown

As water pressures increase back to the full 60m head, the axial forces in the lining to resist this pressure increase dramatically and as a result the lining compresses and the gap at the invert increases to 50mm as shown in Figure 15. However the difference in water pressure between the invert and the crown remains unchanged so there is only a modest increase in bending moment due to the slight increase in ovaling caused by the larger gap at the invert.

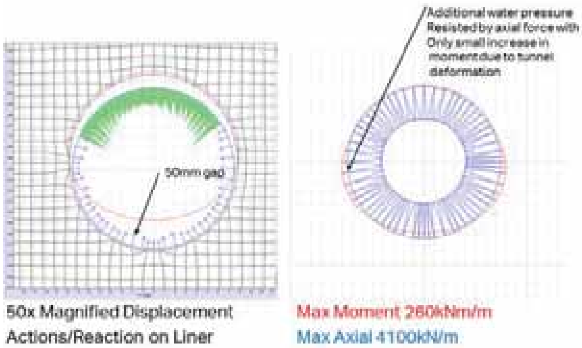


Figure 15: Full reinstatement of water

The effect of the gap in the invert of the tunnel is a major consideration. As shown in Figure 2 even a 5mm wide gap would provide higher permeability in the longitudinal axis of the tunnel than most rock masses. The implications of this are that if there is any leak in the tunnel lining water can flow along the length of the tunnel to supply water to the leak. Conversely if the tunnel is passing through a hill or ridge where the groundwater levels vary along the length of the tunnel then the tunnel will act as a longitudinal drain evening out he water pressures along the tunnel. This may be detrimental to regional groundwater and beneficial uses. To address this, one technique that can be used is to provide annular grout rings at interval along the tunnel that can be grouted after water pressures have reinstated to reduce longitudinal conductivity.

There is a tendency, particularly with highway tunnels of three or more lanes, to see circular tunnels as inefficient use of space because the space required for three lanes of truck traffic and emergency space is wider than it is high. However this needs to be balanced against the much higher lining stresses developed in a non-circular lining. Consider the same ground conditions as above but with an oval shaped tunnel excavation height of 80% of the excavation width. For reasons that will become apparent the excavation dimensions to provide an equivalent traffic envelope to a 15m diameter circular excavation are 17m x 13.6m to allow for a 1.0m thick liner.

The oval shape results in about three times higher bending moments than a circular tunnel under self-weight and as the water pressure rises to the crown, but the most significant difference occurs as water pressures continue to rise to the full reinstated pressure of 60m above the invert. While for a circular tunnel the additional pressure in all directions is equal and hence is resisted completely by axial compression, with an oval tunnel the additional forces due to increasing pressure are always 25% higher in the vertical direction than in the horizontal direction, hence the oval wants to be squashed flat increasing the moments in the liner. The result is that while the axial force in the liner is similar for a circular or oval liner, the moment in the oval liner is almost ten times the moment in the circular liner as shown on Figure 16.

Even increasing the liner to 1m thickness does not give sufficient bending capacity for an unreinforced liner and significant structural reinforcement needs to be added.

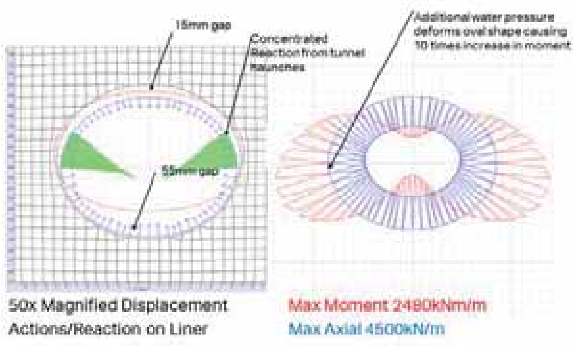


Figure 16: Final stage 17x13.6m tunnel

As it turns out, for this example changing from a circular to an oval shape while maintaining the same traffic envelope for three lanes of traffic as shown in Figure 17 actually increases the volume of excavation spoil, doubles the volume of concrete liner, adds a requirement for additional reinforcement and produces a worse grouting problem for the gap between the liner and rock, hence it is clearly a more expensive solution.

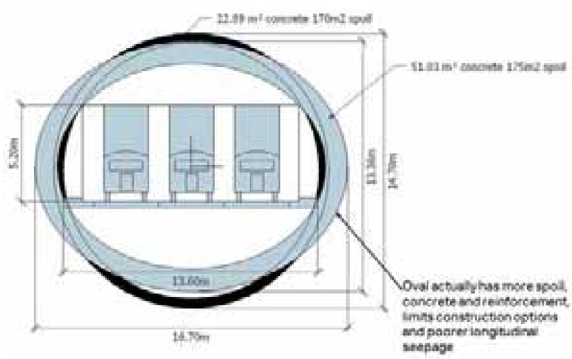


Figure 17: Round versus oval

The above example assumes road header construction with temporary dewatering as this best illustrates the range of issues associated with changes in groundwater pressure during construction. It should be noted that mined construction under compressed air was carried out in parts of Melbourne including the Hobson's Bay Sewer and parts of the Melbourne City Loop which would reduce these impacts. Modern tunnel boring machines (TBMs) use earth pressure balance or slurry techniques and install circular segmental linings with a pressure grouted annulus immediately behind the TBM to maintain a back pressure close to hydrostatic conditions. Using these techniques the potential for large gaps between lining and rock is significantly reduced although some segregation of grout can still cause a longitudinal drainage path to develop along the crown. This can be addressed with additional post-grouting where required.

4.7 Potential for "Damming"

In one sense granular aquifers in the base of paleochannels can be thought of as underground rivers with some potential for groundwater to flow along the length of these aquifers. An important distinction between these aquifers and a river at the surface is that even high permeability aquifers have a transmissivity many orders of magnitude lower than an open river channel, hence the volumes of groundwater flow down these aquifers is comparatively small. Nevertheless, the potential for excavation structures to dam flow in the aquifer does need to be considered. A common example of damming effects is the potential for diaphragm walls installed for cut and cover tunnelling to potentially cut through aquifers in the base of paleochannels causing a difference in groundwater levels either side of the diaphragm wall.

4.8 Drained or Tanked?

The examples in the preceding sections clearly demonstrate that designing structures to resist the forces from water pressure requires a considerable increase in the strength of the structures compared with a structure that is permanently drained. On this basis it could be reasonably expected that the direct up-front capital costs of a tanked (water tight) structure will generally be higher than the up-front costs of a drained structure. However, the up-front costs of the structure are only part of the total costs, particularly if whole of life costs are considered and a dollar amount is assigned to risks of impacts to the structure or third parties.

When weighing up the whole of life costs consideration needs to be given to factors including the following:

- The direct costs over the structure life of installation, maintenance and replacement of pumps and dewatering systems including the costs of de-clogging blocked drainage systems.
- The costs of groundwater disposal over the structure life including the possibility that trade waste agreements change with time.
- The risk to the structure if the drainage system fails.
- If the permanent structure is tanked, can dewatering be avoided for the temporary condition offset against the risks below?
- The impact of dewatering on third parties or the environment (some examples listed below):
- Dewatering may result in significant changes in the direction of groundwater flow either temporarily or permanently which may cause existing contaminated groundwater (if present) to change course. Contaminated groundwater may be drawn into excavations or into areas that are currently not contaminated.
- Dewatering or exposure of Coode Island Silt, Holocene soils and Melbourne Formation with an acid sulphate potential (PASS) can lead to oxidation within temporary excavations. Permanent groundwater lowering in PASS may result in long term change in oxidation conditions and acid generation.
- Settlement caused by dewatering particularly where soft soils are present.

5 CONCLUSION

The examples presented in this paper demonstrate the major impacts that groundwater can have both on deep excavations and on drawdown induced impacts on the surroundings. The author hopes that these examples will serve as guidance for designers in Melbourne.

6 DISLAIMER

The information on hydrogeological conditions in the central Melbourne area contained in Sections 2 and 3 of this paper are based on the references in the public domain as listed below and the author's experiences up to 2014. There has been extensive geotechnical and hydrogeological investigations in the central Melbourne area since 2014 which have not been taken into account and may change some of these interpretations.

7 GLOSSARY

The following definitions are extracted from CIRIA (2000) as a ready guide to the reader of some of the main terms used in this paper. Further definitions and guidance can be found in CIRIA (2000):

- <u>anisotropy</u> - The condition in which one or more of the properties of an aquifer varies according to the direction of measurement;
- <u>aquifer</u> - Soil or rock forming a stratum, group of strata or part of a stratum that is water-bearing (i.e. saturated and relatively permeable);
- <u>aquitard</u> - Soil or rock forming a stratum, group of strata or part of a stratum of intermediate to low permeability which only yields very small groundwater flows;
- <u>confined aquifer</u> - An aquifer overlain by a confining stratum of significantly lower permeability than the aquifer and where the piezometric level is above the base of the confining stratum;

- drawdown - The amount of lowering of the water table in an unconfined aquifer or of the piezometric level in a confined aquifer caused by a groundwater control system;
- leaky aquifer - An aquifer confined by a low permeability aquitard. When the aquifer is pumped, groundwater may flow from the aquitard and recharge the aquifer, (also termed semi-confined aquifer);
- perched water - Water in an isolated saturated zone above the water table. It is the result of the presence of a layer of low or very low permeability above which water can pond;
- permeability - A measure of the ease with which water can flow through the pores of soil or rock;
- recharge boundary – An aquifer boundary that can act as a supply of water to the aquifer;
- storativity - The quantity of water an aquifer releases per unit surface area of the aquifer per unit drawdown (also known as storage coefficient);
- transmissivity - A measure of the ease with which water can flow through the saturated thickness of an aquifer (equal to the product of permeability and saturated aquifer thickness);
- unconfined aquifer - An aquifer, not overlain by a relatively impermeable confining layer, where a water table exists and is exposed to the atmosphere;
- water table - The level in an unconfined aquifer at which the pore water pressure is zero (i.e. atmospheric). Note in a confined aquifer this is equivalent to the phreatic surface, the level to which water would rise in a piezometer.

REFERENCES

Geological Survey of Victoria 1:63,360 Series Melbourne and Ringwood Map Sheets (1974)

W.A. Peck, J.L Neilson, R.J Olds and K.D Seddon (eds) (1992). “Engineering Geology of Melbourne” Balkema 1992, 407pp.

B.J. Hutchison and I.A Lamb (1999). “Construction Management of Hydrogeological Aspects of Melbourne’s City Link Project” Proc. 10th Aust. Tunnelling Conference, Melbourne Vic 1999 pp 35-45

CIRIA (2000) “Groundwater Control Design and Practice” CIRIA Publication C515

M. Ervin, N. Benson, J. Morgan and N. Pavlovic (2006) “Melbourne’s Southbank Interchange - a permanent excavation in compressible clay”, Australian Geomechanics Vol 41 No 3 September 2006, pp 25-40

J. G. Leonard (2006) “Hydrogeology of the Melbourne area”, Australian Geomechanics Vol 41 No 3 September 2006 pp 63-74

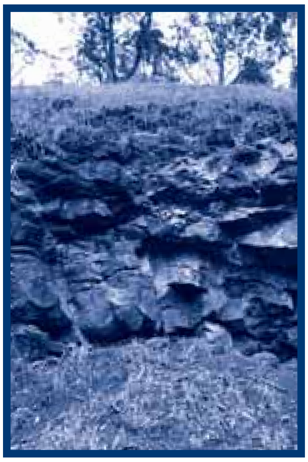
R.A. Day and P. Woods (2007). “Verification of Consolidation Parameters of a Near-Normally Consolidated Clay by Back-Analysis of and Instrumented, Wick-Drained Reclamation” Proc. 10th ANZ Conf. on Geomechanics, Vol. 2, pp 54-58

S.T. Srithar (2010) “Settlement characteristics of Coode Island Silt” Australian Geomechanics Vol 64 45 No 1 March 2010 pp 55-64

D. Paul, S. Barrett, P.Stewart and M. Webster (2014) “The engineering geological evolution of the Jolimont Valley, Melbourne, Victoria”, Australian Geomechanics Vol 49 No 2 June 2014, pp 15-30

AGS VICTORIA 2016 SYMPOSIUM

Excavations and slope stability in Melbourne geology: experiences and recent developments



AUSTRALIAN GEOMECHANICS SOCIETY

VICTORIA CHAPTER